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Cement Stabilized Crushed Stone Base Course Strength and Stiffness Analysis

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ABSTRACT

Approximately 70 – 80% of state departments of transportations (DOTs) utilize the *1993 AASTHO Guide for Design of Pavement Structures* (1993 AASHTO) [1] for structural design of pavements. A growing portion of DOT's are beginning to implement the *Mechanistic-Empirical Pavement Design Guide* (MEPDG), referred to as AASHTOWare Pavement ME [2]. The 1993 AASHTO flexible guide applies empirically derived unitless strength values referred to as structural layer coefficients and structural numbers (*SN*) to pavement thickness and pavement types. The 1993 AASHTO rigid flexible guide utilizes the depth and strength of the concrete pavement, with the composite spring constant (k_c), which is calculated from subgrade resilient modulus, subbase modulus of elasticity (MOE), and the subbase depth. The ME instead includes material properties such as MOE and Poisson's ratio, in addition to a large list of other inputs to provide a pavement design that can resist required traffic loadings and other impacts.

Cement Stabilized Crushed Stone Base Course (CSCSBC) is a subbase alternative that is usually provided to prevent loss of subbase support over time. Often, CSCSBC is designed based on prescribed strength values, regardless of the range of cement contents used. CSCSBC can have varying strength properties that are often not accounted for. The goal of this research was to provide a structural coefficient, spring constant, MOE, and Poisson's ratio based on the actual strength of CSCSBC for use in Arkansas DOT (ARDOT) projects. This research is not focused on the benefits or disadvantages of 1993 AASHTO to MEPDG, and instead will only be focused on the inputs of both.

A range of tests were performed to characterize the strength and stiffness of a test strip located in central Arkansas. The tests conducted were the Static Plate Load Test (StPT), unconfined compression strength, MOE, and Poisson's ratio. The recommended design values for the 1993

AASTHO Guide, the structural coefficient and spring constant, was an $\boldsymbol{a_2}$ minimum of ***0.21*** and $\boldsymbol{k_c}$ ***= 2,000 psi/in***, respectively. $\boldsymbol{a_2}$ would be acceptable to be kept in a range of **0.21-0.30**, based on the 7-day compression strength. For the MEPDG, an equation was provided to relate the MOE to the cement content, and the Poisson's ratio is suggested to be taken from **Table 11-7** from the *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice* [3].

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1.0 INTRODUCTION

Most pavement design in the United States follows two major design disciplines: the *1993 AASHTO Guide for Design of Pavement Structures* (1993 AASHTO) [1] or the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) [4]. The Federal Highway Association (FHWA) conducted a survey in the 1990s of the state Departments of Transportation (DOTs) to determine common design methods. It was found that approximately 80% of states used an AASHTO pavement design procedure, with the majority using 1993 AASHTO [5]. The 1993 AASTHO method utilizes the pavement's existing and future condition and statistical change over time to either calculate an arbitrary value that designates the pavement's required strength for flexible pavement, or calculate a composite spring constant (k_c) for the rigid pavement [1]. This method was purely empirical and was based on a relatively limited dataset [1]. The MEPDG instead uses the structural strength of each layer, as well as the connection and interaction between each layer to find the strength of the pavement [4], combining an empirical evaluation of pavement with a mechanics based strength of materials approach.

Both the MEPDG and the 1993 AASHTO methods were developed in the early 1980s, with future editions of the guides being focused on producing a more accurate and efficient design from the previous edition. Inclusion of existing or newer pavement materials is important to allow designers to utilize the most effective materials for a project. Cement stabilized bases, such as Cement Stabilized Crushed Stone Base Course (CSCSBC), are often an effective base material, but realistic design inputs may not always be available. Traditionally, CSCSBC had one purpose: improve the durability of the subbase at joints of concrete pavements by preventing pumping of fines [6]. But CSCSBC also has improved strength, durability, and flexibility compared to standard aggregate base. Some DOT's pavement design methods may not account fully for the added strength of pavement options, such as CSCSBC [7]. The focus of this research was to provide guidance to help account for the strength of CSCSBC in both the 1993 AASTHO and MEPDG design principles. It should be clarified that this research does not investigate the durability and cracking caused by

In early April of 2019, Weaver-Bailey Contractors, Inc., constructed a CSCSBC test strip in North Little Rock, AR. The test strip was constructed following ARDOT specifications, which requires a cement content of 3-8%. The test strip was 8-inches deep, 300-feet long, and 40-feet wide. The 300-foot length was composed of six 40-foot sections at 3% to 8% cement content in 1% increments, and five 12-foot intermittent sections where cement content increments might have mixed. Each full 1% increment section (40-foot sections) was tested by the standard plate load test (StPT) on site, and then 3 cores were removed from each section to be tested for unconfined compression strength, modulus of elasticity (MOE), and Poisson's Ratio. ARDOT specifies a minimum 7-day compressive strength of *750 psi* for CSCSBC and does not specify a structural number in the *ARDOT Roadway Design Plan Development Guidelines* [7]. The goal of this work was to establish better accuracy in the material properties of CSCSBC for ARDOT. A combination of field testing, laboratory testing, and analytical work was performed to provide simple design recommendations.

2.0 BACKGROUND

2.1 Cement Stabilized Crushed Stone Base Courses

CSCSBC's main purpose is to provide stabilization and increase stiffness of the pavement, as is the purpose of any stabilized base. The stiffness of the pavement structure changes the way the load is distributed. A higher stiffness distributes the load further away from the point of application, while a more flexible pavement results in a higher load concentration [5]. Several

DOTs currently neglect or underutilize the structural support CSCSBC provides [8]. ARDOT specifies a required 3-8% Type I cement content by weight as well as requiring the gradation to much ARDOT's Class 7 specifications [See **Table 4**] [9].

The increased stabilization from CSCSBC also serves to counteract the movement of fines and particles in the pavement due to cyclic loading. Near the joints of pavements (edge of flexible pavements and relief joints in rigid pavement), tire loads cause a deflection. Once the tire exits the initial slab and moves to the second slab, the initial slab rebounds, creating a negative pressure zone underneath that joint (**Figure 1**). This, coupled with the positive pressure underneath the newly deflected second slab, pushes water out from under the second slab and pulls water out from under the first slab's subbase into the empty joints. An unstabilized base will allow fines to be pulled along with the moving water. At first, this loss of fines is not significant, but after tens of thousands of loadings, the base at the joints can begin collapsing due to loss of considerable amounts of fines, causing severe joint cracking or faulting [10]. A stabilized base minimizes or eliminates fines transfer [5].

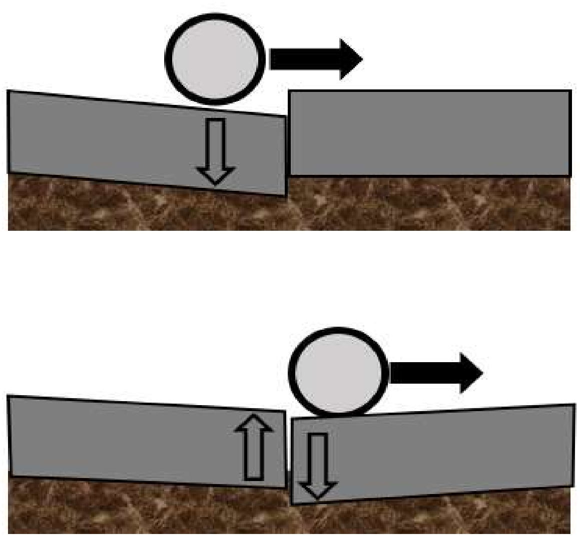

Figure 1: Cyclic loading of joints

2.2 1993 AASHTO Design Method

Because this study's goal was to provide realistic pavement design inputs for CSCSBC, a discussion of the prevailing design methods and how the strength of CSCSBC affects the resulting design is warranted. The following sections discuss the 1993 AASTHO method (the prevailing method in use in Arkansas) and the MEPDG method. A sensitivity analysis is shown for the 1993 AASHTO method to illustrate how a change in CSCSBC design inputs would affect rigid and flexible pavement thicknesses. No sensitivity analysis was conducted for MEPDG due to MEPDG already providing a much more in-depth analysis of pavement design compared to the 1993 AASTHO method and it already accounted for MOE and Poisson's ratio for cement stabilized base courses [4]. Instead, the testing would provide a more accurate representation of the material.

The 1993 AASHTO design method utilizes two main equations, **Equation 1** and **2**, to develop the structural number (*SN*) and concrete slab thickness (*D*) through iteration and back-calculation. The equations were created based on the results of AASHO Road Test [11]. Flexible design is seen in **Equations 1** and **3,** while rigid design is shown in **Equation 2** [1].

$$[\text{Equation 1}]: \log_{10} W_{18} = Z_R * S_0 + 9.36 * \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40+\frac{1094}{(SN+1)^{5.19}}} + 2.32 * \log_{10}(M_R) - 8.07$$

$Where$:

$W_{18} = \#\ of\ 18\ kip\ Equavilent\ Single\ Axle\ Loads\ (ESAL)\ for\ flexible\ pavement$

$Z_R = Standard\ Normal\ Deviate$

$S_0 = Overall\ Standard\ Deviation$

$\Delta PSI = Allowable\ Servicability\ Loss\ at\ End\ of\ Design\ Life$

$M_R = Subgrade\ Resilient\ Modulus$

$SN = Structural\ Number$

[Equation 2]: $\log_{10} W_{18} = Z_R S_0 + 7.35 * \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.5-1.5}\right)}{1.00+\frac{1.64*10^7}{(D+1)^{8.46}}} +$

$(4.22 - 0.32 * p_t) \log_{10} \frac{S_c * C_d * (D^{0.75} - 1.132)}{215.63 * J * \left(D^{0.75} - \frac{18.42}{\left(\frac{E_c}{k_c}\right)^{0.25}}\right)}$

$Where:$

$W_{18} = \#\ of\ 18\ kip\ Equavilent\ Single\ Axle\ Loads\ (ESAL)\ for\ rigid\ pavement$

$Z_R = Standard\ normal\ deviate$

$S_0 = Overall\ standard\ deviation$

$\Delta PSI = Allowable\ servicability\ loss\ at\ end\ of\ design\ life$

$p_t = Terminal\ serviceacbility$

$k_c = Composite\ Modulus\ of\ subgrade\ reaction\ (\frac{psi}{in})$

$S_c = Portland\ Cement\ Concrete\ modulus\ of\ ruptures\ (psi)$

$E_c = Portland\ Cement\ Concrete\ modulus\ of\ elasticity\ (psi)$

$J = Empirical\ joint\ load\ transfer\ coefficient$

$C_d = Empirical\ drainage\ coefficient$

$D = Required\ PCC\ slab\ thickness\ (in)$

[Equation 3]: $SN = a_1 * d_1 + \sum_{i=2}^{n} a_i * d_i * m_i$

$Where:$

$SN = Structural\ number$

$a_1 = Structural\ coefficient\ of\ surface\ layer$

$d_1 = Thickness\ of\ surface\ layer$

$a_i = Structural\ coefficient\ of\ consecutive\ layers$

$d_i = Thickness\ of\ consecutive\ layers$

$m_i = Drainange\ coefficient\ of\ layers$

$n = Final\ layer\ of\ pavement\ (usually\ subgrade)$

2.2.1 1993 AASTHO Design Inputs

The 1993 AASTHO design inputs can be determined from a range of sources; from traffic analysis, decided by state DOT officials, or calculated from the actual pavement design. **Equation 1** and **2** are used to back-calculate the values *SN* and *D*. These two values define the total pavement strength due to each pavement layer (for flexible pavements) or the required thickness of the rigid concrete pavement, respectively. The important design inputs are described below:

- $\boldsymbol{Structural\ Number\ (SN)}$: The arbitrary structural strength of the total pavement. This is based on the subcomponents of the roadway's structural coefficient and thickness. For example, the standard Arkansas DOT's (ARDOT) asphalt concrete hot-mix (ACHM) surface has a prescribed structural coefficient (a_1) of 0.44 [7], which when multiplied by the depth of the section gives the *SN* increment of that layer. *SN* is usually iterated through Equation 1.
- $\boldsymbol{D}$: the depth of the concrete pavement. This is the value that is desired from Equation 2, usually achieved through iteration.
- $\boldsymbol{M_R}$: defines the subgrade's dynamic strength due to high cyclic impact loads. This value is mostly important for flexible pavement design but is also used in rigid pavement design for calculating the modulus of subgrade reaction, k.

- $\boldsymbol{k}$: the modulus of subgrade reaction is the expected spring constant (or stiffness)of the subgrade. This is based on the subbase thickness, subbase MOE, and the resilient modulus of the subgrade [5].
- $\boldsymbol{E_c}$: MOE of the concrete pavement. Measured or based on the concrete compressive strength.

2.2.2 Sensitivity of 1993 AASHTO Pavement Design to Subbase Inputs

Due to the way the rigid and flexible design equations are constructed in 1993 AASHTO method, the strength and depth of subbase affects the design in significantly different ways. The flexible design calculates a *SN* for the entire pavement section while the rigid design focuses entirely on the depth of the concrete. To illustrate the effects of changing the subbase input values on the resulting pavement design (both flexible and rigid) a sensitivity analysis was performed for a typical Arkansas highway. The analysis was performed as follows: to calculate the required ESALs, a *SN* or *D* value must first be assumed then using the AASHTO Load Equivalency Factor (LEF) [1] tables to find ESAL values. Once a new *SN* or *D* is calculated, the pavement is redesigned using this new value, resulting in an iterative design process.

Table 1: Equivalent Single Axle Load (ESAL) design inputs.

ESAL DESIGN INPUTS		
2019 AADT	33,000	Vehicles Per Day (vpd)
2039 AADT	45,000	Vehicles Per Day (vpd)
Percent Trucks	15%	%
Two-way Factor	0.5	N/A
Lane Distribution Factor	0.8	N/A
Design Lifetime	20.0	Years
Initial Structural Number Assumption	4.0	N/A
Initial Concrete Depth Assumption	9.0	Inches
Growth Factor of traffic	23.27	N/A
Growth Rate of traffic	1.56%	%
Reliability of Roadway	85%	%

Table 1 contains design inputs used for both the rigid and flexible sensitivity analysis. These inputs are based on a previous ARDOT Job: Job 100959. Job 100959 was an asphalt/concrete alternative bid job for Hwy. 63B – Hwy. 18 (S) in Craighead County, Arkansas [12]. The concrete alternative utilized CSCSBC subbase but assumed the subbase had equivalent MOE as Class 7. Both alternative designs must have equal depth and meet all ARDOT specifications [7]. This analysis indicates the effect of subbase input values but does not fully evaluate multiple variable sensitivity. **Table 2** shows suggested design values for flexible pavement and rigid pavement from the *ARDOT Roadway Design Plan Development Guidelines* [7]. This document recommends design values for different pavement materials. Importantly, a_4, the structural number for the subbase is *0.14* for compacted class 7 aggregate. ARDOT does not provide a structural number for CSCSBC [7].

Table 2: AASHTO inputs from *Arkansas Roadway Design Plan Development Guidelines*

Flexible Pavement Inputs		**Rigid Pavement Inputs**	
W_{18}	46,0.37,847	W_{18}	71,497,927
Z_R	-1.036	Z_R	-1.036
S_0	0.45	S_0	0.35
p_i	4.5	p_i	4.5
p_t	2.5	p_t	2.5
ΔPSI	2.0	ΔPSI	2.0
M_R	5,000 psi	M_R	5,000 psi
a_1 (ACHM Surface)	0.44	f'_c	4,000 psi
a_2 (ACHM Binder)	0.44	S_c	474 psi
a_3 (ACHM Base)	0.36	E_c	3,604,997 psi
a_4 (Class 7)	0.14 [7]	J	3.2
d_1 (ACHM Surface)	2 in.	C_d	1.0
d_2 (ACHM Binder)	4 in.		
d_3 (ACHM Base)	5 in.		
d_4 (Subbase)	8 in.		
m_i	1.0		

Table 3: Initial pavement inputs for flexible pavement [7]

Type	Depth (in)	Structural Coefficient	*SN*
ACHM Surface	2.0	0.44	0.88
ACHM Binder	4.0	0.44	1.76
ACHM Base	5.0	0.36	1.80
TOTAL	11.0	N/A	4.44

Figure 2 shows the sensitivity analysis for a flexible pavement based on 1993 AASHTO method. Based on the design inputs given in Tables 1, 2, and 3, the required *SN* value is 5.8. All other layers being equal, the required structural coefficient of the subbase was 0.17. For subbase layer coefficients in the typical range (0.10-0.24), the total *SN* of the pavement can range from 5.24 to over 6.0. Underestimating the strength contribution of the subbase layer can thus have a significant effect on the required depth of the other layers

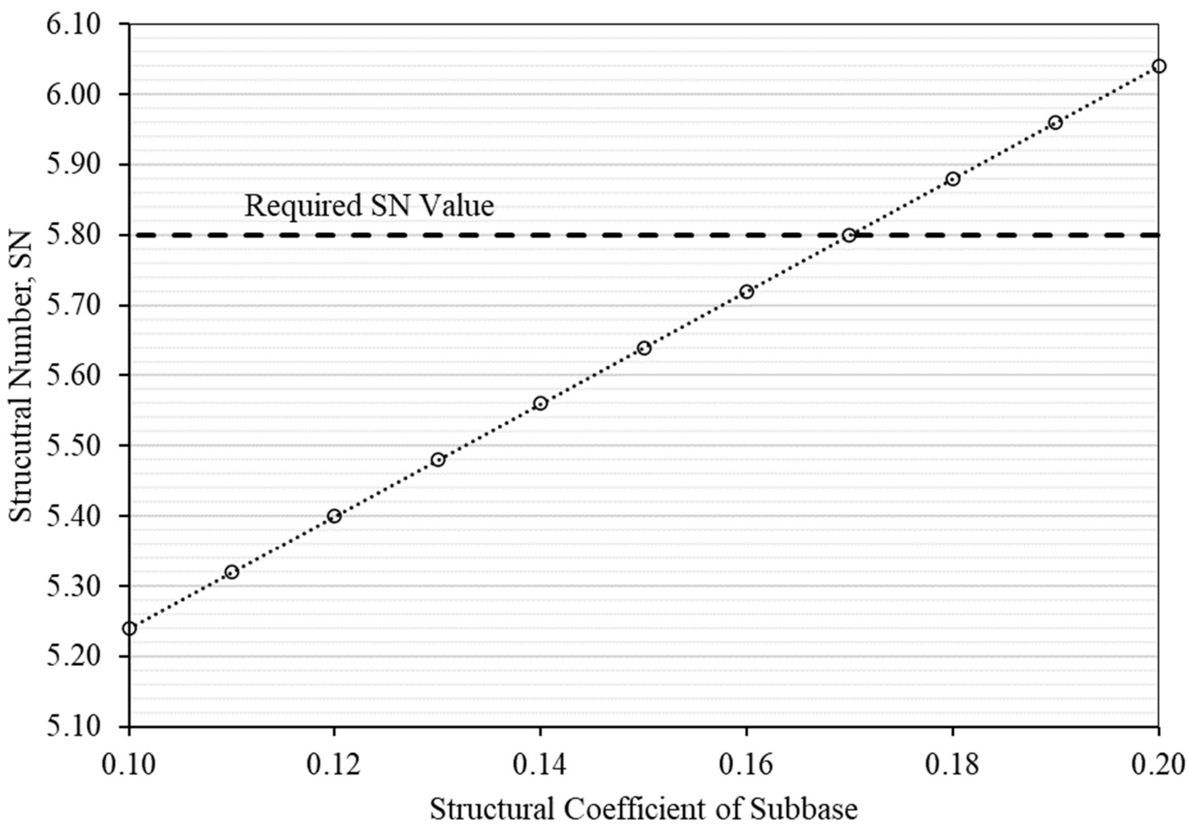

Figure 2: Sensitivity analysis of flexible pavement to the structural coefficient of subbase

Figure 3 shows the concrete rigid pavement sensitivity analysis based on the same design inputs used in the flexible analysis (Tables 1 and 2) but including three separate subbase MOE of *30,000*, *200,000*, and *1,000,000 psi*. Each of the four points on **Figure 3** for each subbase MOE are based on different subbase depths, *6*, *9*, *12*, and *15 inches*, respectively. These points appear in descending order on the figure. Class 7 subbase is designed for a *30,000 psi* MOE by ARDOT [7]. No MOE design value is given for CSCSBC. A modest increase in subbase MOE from 30,000 psi to 200,000 psi results in a decrease of at least *0.25 in.* in required concrete pavement depth. Clearly, CSCSBC will have a greater modulus than crushed stone base, therefore this increase in strength should be reflected in the design depth of the concrete pavement. If the MOE for CSCSBC is *1,000,000 psi*, the depth of concrete pavement can be reduced by nearly 1 in.

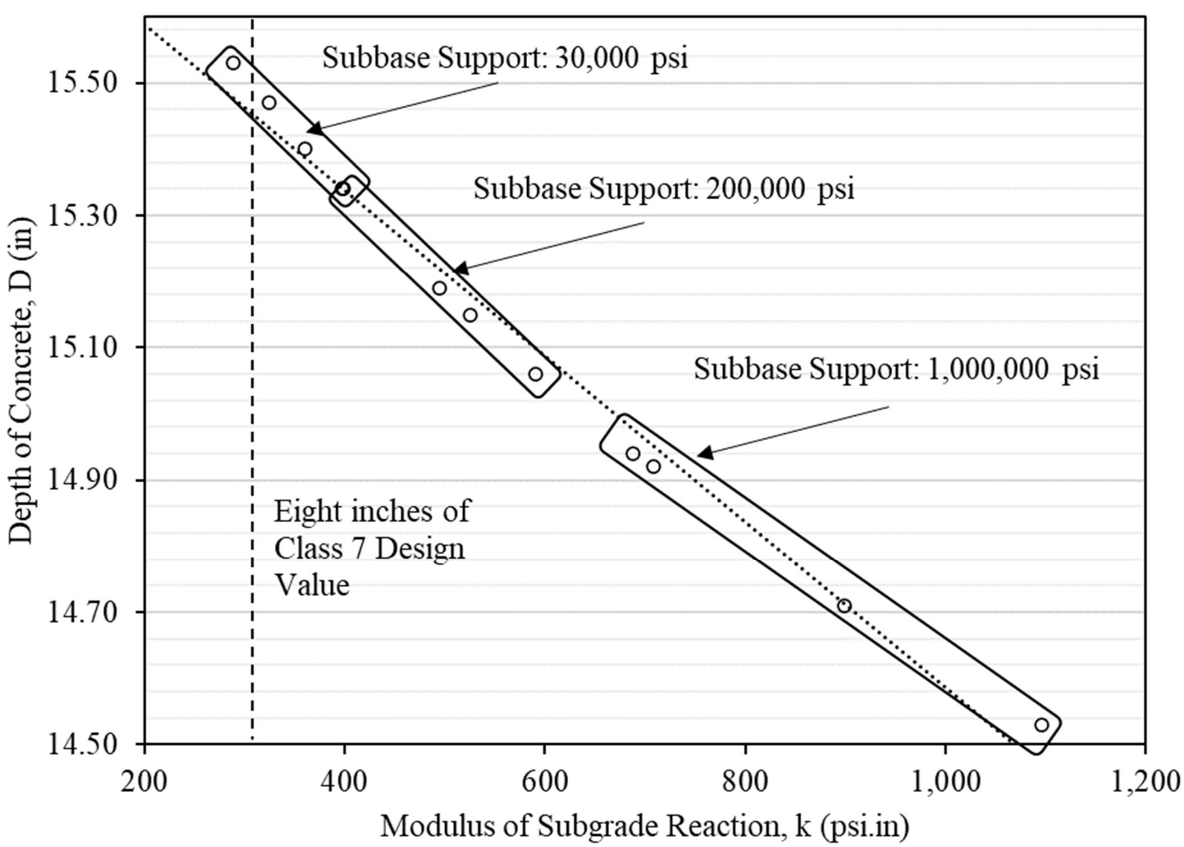

Figure 3: Sensitivity of required concrete depth to modulus of subgrade reaction

2.3 MEPDG

The MEPDG is fairly similar to the 1993 AASHTO in terms of the initial pavement inputs. However, the MEPDG requires a larger number of environmental inputs overall including traffic volume, climate conditions, subgrade and existing pavement conditions [4]. MEPDG utilizes software to design the pavement such as *AASHTOWare Pavement ME Design* [2], compensating for the interaction between each pavement layer, the pavement's strength, and the effects the environment and traffic will have on the pavement over time. The more information and data available, the better the model will conform to real-life behavior [4].

Therefore, to provide a more accurate representation of CSCSBC for the MEPDG, a Poisson's ratio and MOE analysis was conducted. *A Manual of Practice* on the MEPDG suggests estimating the Poisson's ratio and directly calculating the modulus of elasticity [3], so the research will go further than the MEPDG guides suggest. As mentioned before, no sensitivity analysis was conducted on MEPDG due time constraints and MEPDG accounting for subbase material more accurately compared to the 1993 AASHTO method [4].

3.0 Materials and Procedures

As mentioned before, the input values for subbase material for both the 1993 AASHTO and the MEPDG revolve around the static and dynamic strength values of the pavement material. 1993 AASHTO uses the cyclic resilient modulus for rigid pavement design, while the flexible design structural coefficients are either based on the compressive strength or MOE. The MEPDG requires the MOE and the Poisson's ratio of the subbase material. There are many variables that affect CSCSBC material strength: level of compaction, water-to-cement ratio, cement content, age at time of testing, and aggregate strength. To collect results that accurately represented in-field conditions, a local paving contractor, Weaver-Bailey Contractors, Inc., constructed a large test strip of CSCSBC to perform a range of field and lab tests.

The strip was 300 by 40 ft, with an average depth of 8-in., constructed according to ARDOT specifications [9]. The strip was in Central Arkansas, near the city of Jacksonville. Construction was started on April 19th and completed on April 20th, 2019. The 300-ft strip was composed of six 40-foot sections and five mixed 12-foot sections, each 40-foot section containing a different cement content, ranging from 3 to 8% in 1% increments. On-site testing and core procurement were completed on May 21st, 2019 (31 days since the section was placed). The location of the strip is shown in **Figure 4**.

Figure 4: Location of test strip in Central Arkansas and picture from construction day

The site of the test trip was a storage area for paving equipment and moveable concrete barriers. As such, the subgrade was well compacted from the movement and storage of heavy equipment. The CSCSBC was mixed in a concrete central mixing plant nearby and delivered by dump trucks. The site was compacted and graded according to standard practice for ARDOT roads. To ensure testing was performed on sections of a single cement content, overlapping areas (12-foot sections) were marked with spray paint and testing was avoided within those sections. Field testing and coring was performed between the marked areas.

The aggregate used in the CSCSBC was quarried in El Paso, Arkansas and was a crushed limestone with a 1 ½ in. nominal maximum aggregate size. This same quarry was sampled for the aggregate used in the laboratory specimen testing. The aggregate gradation of the sample is shown in **Table 4**. The aggregate was taken to the laboratory in an "as-received" condition from the quarry operators. The gradation was conducted according to ASTM D6913 [13]. Compared to a Class 7 gradation, this gradation has a lower percentage of fines passing than is allowed in the #200 sieve and the #4 and 3/8" sieve were overloaded in an 8" and 12" sieve analysis. The laboratory specimens were compacted with this material, nonetheless, because the material is representative of the in-place material from the testing strip. The fines within the CSCSBC sample are bound due to the cement in the material and this decrease in fines will not be of critical importance; the main strength of CSCSBC is dependent on the cement content and the course aggregate [10]. A type I portland cement was used at the stated replacement rates (3-8%) from total weight (including water) according to ARDOT specifications [9].

Table 4: CSCSBC aggregate gradation

Sieve	ARDOT **Specifications** [9]	**Gradation Report**
	% Passing	
3”	100	100
2”	100	100
1 ½”	100	100
1”	60 – 100	92
¾”	50 – 90	75
3/8”	–	44
#4	25 – 55	29
#10	–	19
#40	10 – 30	11
#200	3 - 12	**1**

According to the batch tickets, the actual cement contents, and water to cement ratios (*w/c*) of the test strip are shown in **Table 5**. Differences between the targeted cement content are due to the equipment used to load the mixer at the batch plant. In a typical CSCSBC placement, *w/c* may not be of primary importance to the contractor, however the *w/c* should be expected to have an influence on the strength of CSCSBC [10]. Since this was a field test performed with actual construction equipment and methods, complete control over exact quantities and proportions was not possible. Cement can be lost during loading, placement, and compacting so variability is to be expected. The variability in actual cement contents and *w/c* is assumed to be typical for real pavement construction projects and the impact of this variability is not expected to be of significant concern over providing a good product

Table 5: Actual cement contents and water to cement ratios of test strip batches

Targeted Cement Content	Actual Cement Content	Actual w/c
8.0%	7.7%	0.613
7.0%	6.8%	0.545
6.0%	5.9%	0.470
5.0%	5.0%	0.467
4.0%	3.9%	0.538
3.0%	3.0%	0.665

On-site testing included one static plate load test (StPT) [14] and three nuclear density tests (see **Table 6**) [15] for each of the six 40 ft sections. Three 4-inch diameter cores were also pulled from each section to bring back to the laboratory for further testing. The cores were sealed in plastic bags as soon as they were removed from the pavement. All three cores were weighed and measured to determine a density once back in the laboratory. Two of these cores were used to measure the MOE [16] of the CSCSBC, and then all three were tested to determine the uniaxial compression strength [17]. Falling weight deflectometer testing was performed on site, but the results were difficult to interpret due to the lack of a wearing surface layer.

Table 6: Nuclear and core density results from test strip

Cement Content	Dry Density (lb/cf)	Wet Density (lb/cf)	Core Density (lb/cf)
8.0%	136.5	143.5	141.4
7.0%	138.9	146.5	146.3
6.0%	133.9	141.2	141.6
5.0%	121.9	129.2	145.1
4.0%	132.8	141.3	148.1
3.0%	136.1	145.4	146.1
Average	133.3	141.2	144.7

The StPT is completed by loading a specific diameter plate on the surface of a pavement to a specific load. Once the load is achieved, the pressure is released, and then reloaded three times [14]. The last three load patterns usually achieve a linear deflection vs. pressure graph. These lines are then used to approximate a spring constant (k) by taking a specific pressure, usually *10 psi* [6], drawing a horizontal line to the pressure vs. deflection lines, and then going vertically to the axial deflection. For this work, load was applied with a pneumatic cylinder and the load and deflections were measured using a load cell and two linear voltage differential transformers (LVDTs). An example of the StPT set up is shown in **Figure 5.**

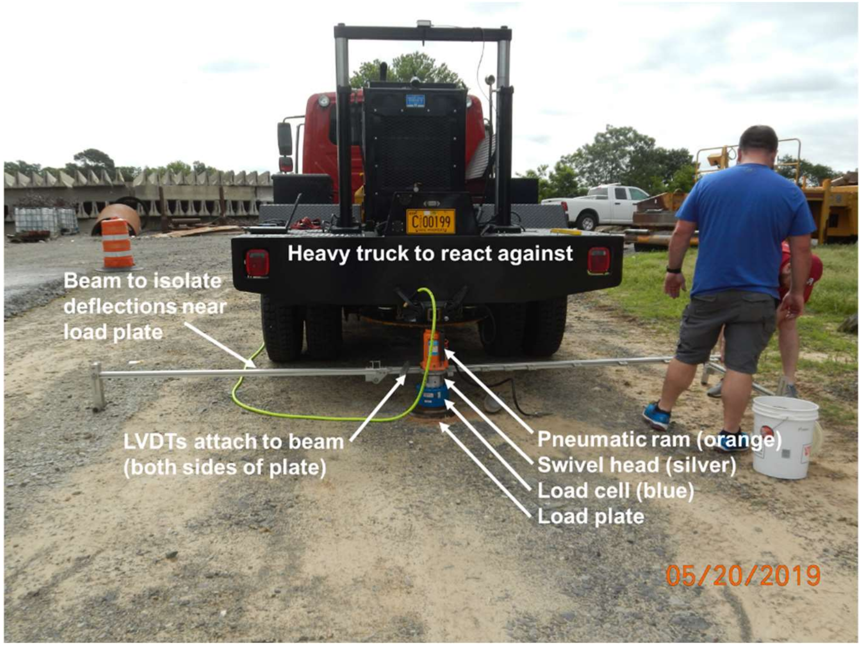

Figure 5: StPT set-up on-site

Finally, some of the results from the core testing showed unusually high values for modulus of elasticity and compression strength. It is possible this was related to the variable *w/c* of the test strip sections. To verify the field results against more controlled laboratory samples, a new set of samples were molded and compacted to match the original core's in terms of cement content, aggregate type, density, and compaction. Standard compaction was performed using a modified proctor [18] mold and hammer according to ASTM D698 [19]. These were tested using a compressometer for MOE and Poisson's ratio, and then were tested to failure in compression. Mixture proportions for these samples are given in **Table 7**. Variability in cement content is caused by small increases in water to make the sample workable and safe to remove from mold.

Table 7: Proportions for laboratory CSCSBC samples

Cement %	3.0%	3.9%	4.9%	5.9%	6.9%	7.9%
Water (lb)	4.02	3.96	4.15	4.06	4.07	3.77
Cement (lb)	1.81	2.42	3.02	3.63	4.23	4.84
Rock (lb)	55.67	55.12	54.57	54.07	53.45	52.89
Total (lb)	61.50	61.50	61.75	61.75	61.75	61.50

4.0 RESULTS

The results of the testing are broken up into two sections: results of the field testing performed at the CSCSBC pad and the results from samples created in the laboratory. For the 1993 AASHTO guide the modulus of elasticity, compression strength, and static plate load test (StPT) of the material were related to the structural coefficient and the StPT itself was used to calculate a spring coefficient for each section. For the MEPDG, results of the modulus of elasticity and Poisson's ratio were used to approximate pavement designs.

4.1 Results of Field Testing

Testing performed on-site at the test strip included FWD, nuclear density, and StPT tests. Cores were taken from the test strip and compressive strength, MOE, and density was measured from these cores. FWD tests were inconclusive due to the lack of a surface layer, so these results are omitted.

4.2.1 Results of Density Testing

Density testing was performed on cores taken from the test strip and from nuclear gauge testing. The results of the density tests were summarized in **Table 6.** Variability in density can be attributed to gradation, placement, and compaction of the material on-site and during forming of cores.

4.2.2 Static Plate Load Test Results

The StPT was performed on each section of the test strip. The 3% section had unusual results due to subgrade failure. In the 7% section, inconsistent results led to the test being repeated several times. In other sections the results of subsequent loadings were similar to the initial loadings, therefore the data was considered to be adequate. In lieu of showing all StPT data (included in **Appendix**), the results from the 5% section are shown in **Figure 6**. While a reference pressure of *10 psi* is usually recommended to develop a stiffness from the StPT, it can be seen the lowest pressure value in which the best-fit lines are still linear was *20 psi.* Using *10 psi* would require the best-fits lines to be extrapolated where data was not collected. Therefore, a value of *20 psi* was instead chosen to select a stiffness. This chosen pressure was divided by the axial deflection to give a *k* value expressed as *psi/in*. This procedure is recommended in the literature [6].

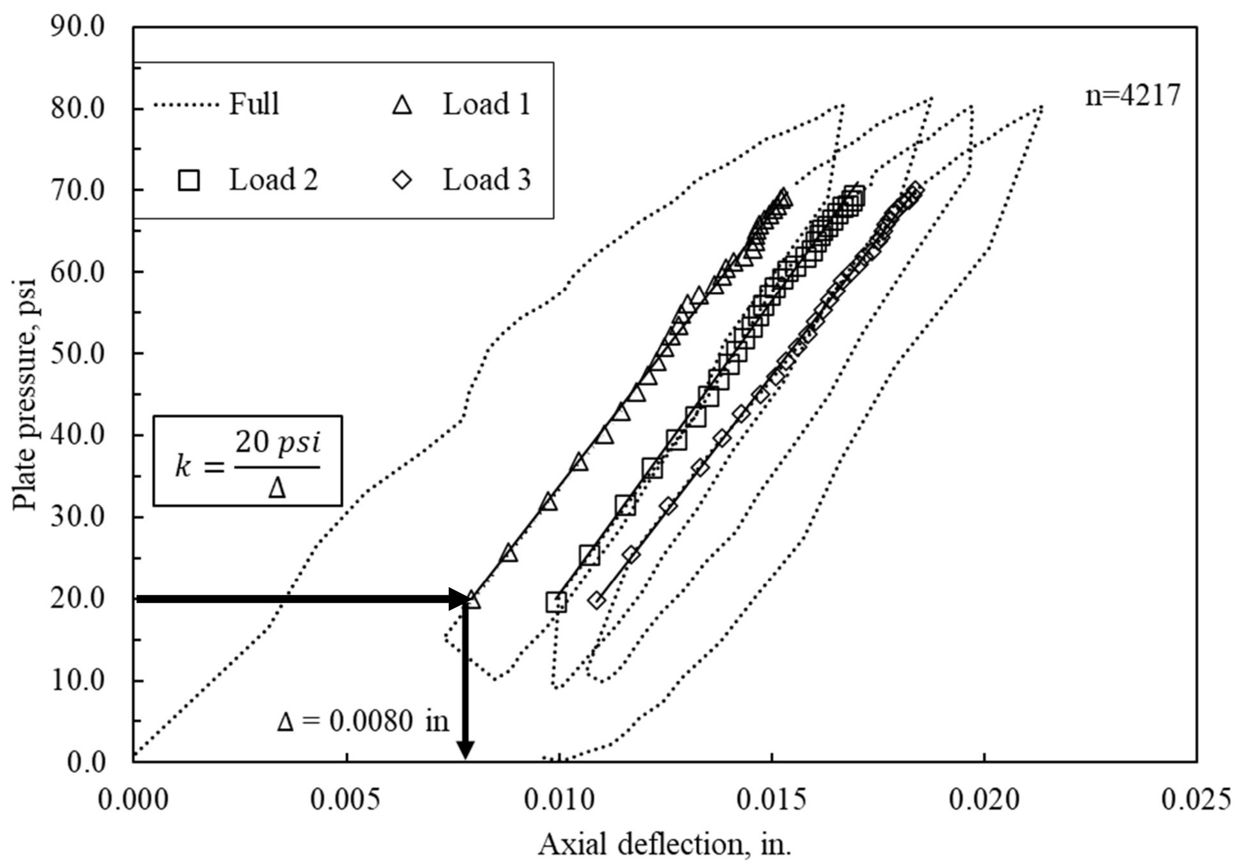

Figure 6: Static Plate Load Test results for 5% cement section

These results are summarized in **Table 8**. It should be noted the 3% cement content section values were inconsistent compared to the other data and outside of expected parameters for static plate load results. Therefore, the data was omitted in further calculations. The results here could be influenced by the inconsistencies in *w/c* or by the subgrade, which as mentioned before was likely heavily compacted in many locations from equipment storage.

Table 8: Spring constant summary of Static Plate Load Test results

Pavement Section	Pressure: *20 psi*
Subgrade	236 psi/in
3% Cement	*23609* psi/in
4% Cement	1404 psi/in
5% Cement	2,118 psi/in
6% Cement	4,618 psi/in
7% Cement	1,749 psi/in
8% Cement	1,601 psi/in
Average of 4-8%	**2,298** psi/in

4.2.3 Compressive strength and MOE from Field Cores

Field cores were removed from the CSCSBC test strip and MOE testing was performed on the specimens followed by compressive strength testing. The total average length of the specimens was only 6 in., as the bottom of the cores would break off during drilling, so the dimensions do not conform to a 2:1 length to diameter ratio typical for concrete cylinder testing. The results of the MOE and compressive strength tests are shown in **Table 9.** These values showed a lot of variability (4% has highest MOE and second highest compression strength). This variability has many possible explanations, from *w/c*, compaction, and/or cement content. To verify these results, further testing was conducted on in-lab produced cores.

Table 9: Compression strength and modulus of elasticity data from cores

Cement	Compression Strength (psi)	Average (psi)	Modulus of Elasticity (psi)	Average (psi)
3%	886	1,090	474,593	1,409,700
	903		1,446,559	
	1,489		2,307,918	
4%	2,535	2,260	3,672,204	2,637,400
	2,527		2,957,684	
	1,702		1,282,324	
5%	1,777	1,570	2,038,276	2,371,700
	1,529		2,195,167	
	1,415		2,881,530	
6%	1,812	1,980	1,341,109	1,989,200
	1,724		2,381,505	
	2,398		2,245,107	
7%	2,078	2,260	2,606,240	2,496,400
	2,118		2,221,858	
	2,591		2,661,080	
8%	2,285	2,270	2,532,748	2,504,200
	2,433		2,734,246	
	2,078		2,245,679	

4.2.4 Compressive Strength and MOE from Lab Cylinders

Because of inconsistencies in the *w/c* and the results of field tests, laboratory specimens were made to provide additional data. The laboratory specimens consisted of cylinders and two of each were tested for compressive strength, MOE, and Poisson's ratio. The results of this testing are reported in **Table 10**. Reasonable results were obtained for compressive strength and MOE, with increasing compressive strength and MOE as cement content increased. At 8% cement content, the MOE, and compressive strengths approach that of a very weak concrete mixture. At 4% (the most used cement content in CSCSBC in Arkansas) the average strength did not meet the specified minimum at 7-days of 750 psi. The MOE was higher than may typically be assumed for CSCSBC, however.

Table 10: Compression strength, MOE, and Poisson's ratio from 7-day samples

Cement	Compression Strength (psi)	Average (psi)	Modulus of Elasticity (psi)	Average (psi)	Poisson's Ratio	Average
3%	416	420	512,751	512,750	0.33	0.33
	-		-		-	
4%	384	390	956,180	689,770	0.39	0.36
	392		423,355		0.33	
5%	650	720	1,465,439	1,358,310	0.14	0.14
	786		1,251,182		0.14	
6%	887	940	1,421,867	1,611,040	0.11	0.07
	995		1,800,219		0.03	
7%	1,116	1,230	2,583,039	2,312,160	0.21	0.22
	1,340		2,041,279		0.23	
8%	1,689	1,640	2,959,227	3,037,870	0.15	0.21
	1,595		3,188,5515		0.26	

5.0 DISCUSSION AND SELECTION OF DESIGN INPUTS FOR CSCSBC

Based on the available field and lab data, reasonable approximations of design inputs for CSCSBC can be recommended. The following discussion combines the results of the testing reported here with correlations and relationships published for pavement design. In combination, these provide a rational way to select design values for CSCSBC for either the 1993 AASHTO method or MEPDG.

5.1 Recommendations for 1993 AASHTO

NHI-05-037 is a Federal Highway Association (FHWA) publication *Geotechnical Aspects of Pavements Reference Manual* [5]. *NHI-05-037* is an extensive summary of the 1993 AASHTO and the developments of the guide in the past 20 years. *NHI-05-037* models exist to calculate the subbase's composite spring constant based on the subbase thickness, MOE, and the subgrade resilient modulus (M_R) [5]. This is shown in **Equation 4** below. The StPT results from Section 4.2.2 resulted in a *k* value for each cement content. **Equation 5** is a simple relationship from *NHI-05-037* between the subbase MOE and spring constant [5]. An MOE for each cement content calculated previously was used to calculate its corresponding *k* value. Due to the variability of the subgrade and the lack of information collected thereof, an overall average subgrade M_R was calculated using the results of the StPT and MOE. This calculated M_R was *25,000 psi* (Standard deviation = *15,000 psi*).

[Equation 4] $ln\, k_c = -2.807 + 0.1253\,(ln\, D_{SB})^2 + 1.062\,(ln\, M_R) +$

$0.1282\,(ln\, D_{SB})(ln\, E_{SB}) - 0.4114\,(ln\, D_{SB}) - 0.0581\,(ln\, E_{SB}) - 0.1317\,(ln\, D_{SB})(ln\, M_R)$

$Where: k_c = Composite\ Modulus\ reaction\ (\frac{psi}{in})$

$D_{SB} = Subbase\ thickenss\ (in)$

$M_R = Subgrade\ Resilient\ Modulus\ (psi)$

$E_{SB} = Subbase\ elastic\ modulus\ (psi)$

[Equation 5] $k_c = \frac{E_{SB}}{19.4}$ Or $E_{SB} = 19.4 * k_c$

This new M_R was then used in **Equation 4** to calculate a k_c for the entire CSCSBC strip. Then **Equations 5** and **6** were used to calculate a structural layer coefficient. **Equation 6** is the structural layer coefficient equation for cement treated based courses from *NHI-05-037* [5].

[Equation 6] $a_2 = 0.249 * \log_{10} E_{BS} - 0.977$

$Where: a_2 = Structural\ layer\ coefficient$

$E_{SB} = Resilient\ modulus\ of\ subbase\ (psi)$

Base on this calculation, an average structural layer coefficient of $\boldsymbol{a_2 = 0.19}$ is found. **Figure 17** in the **Appendix** is a nomograph representing **Equation 4** from *NHI-05-037***.** The maximum *k* in this nomograph is *2,000 psi/in* [5]. Based on the results from StPT testing, **Equation 4 – 6**, and material testing: $\boldsymbol{k_c = 2{,}000\ psi/in}$. This increases the "credit" CSCSBC gets for its improved strength compared to Class 7 but is still within the limits of existing relationships.

In *NHI-05-037* there is a set of graphs and tables that correlate modulus of elasticity and compression strength directly to the structural number. These figures were created using data from testing in Illinois, California, New Mexico, Wyoming, and Texas [5]. These relationships were compared to the strength and MOE data collected from lab samples and field cores shown in **Tables 9** and **10**, respectively. **Figures 7** and **8** show the *NHI-05-037* structural coefficient models for cement treated base course. These models relate the expected 7-day modulus of elasticity and the compression strength to a layer coefficient value. **Figure 7** shows these *NHI-05-037* relationships for the field cores tested at 63-days. Referring to **Figure 7**, the range of structural coefficients based on the compression strength (average) was 0.266 – 0.450 (0.393). The range of structural coefficients based on modulus of elasticity was 0.339 – 0.475 (0.434). These values are high for structural coefficients of subbase material. As indicated previously though, these correlations from the *NHI-05-037* were intended for 7-day strengths, while the field cores were 63-days old at the time of testing.

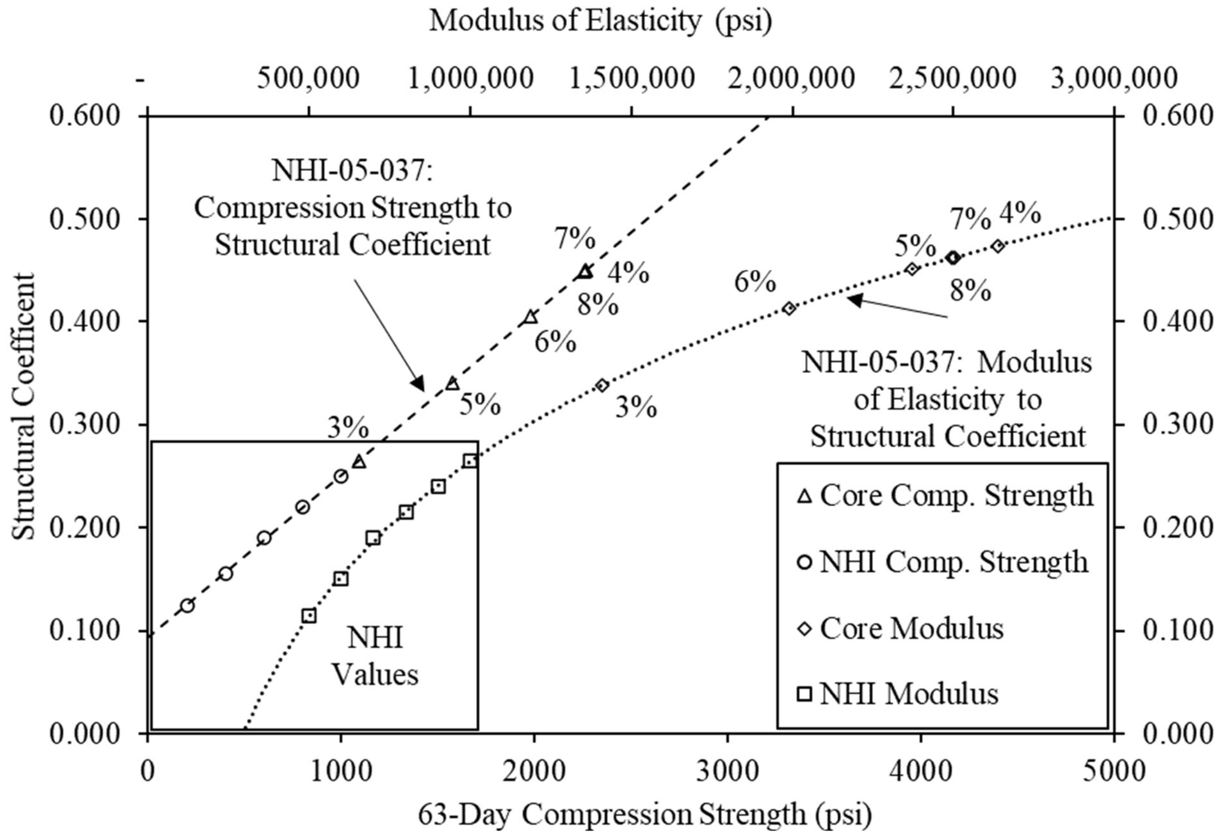

Figure 7: Model of *NHI-05-037* structural coefficient values based on 63-day testing

The in-lab recreations were tested for MOE, compressive strength, and Poisson's ratio at 7 days of age. Strengths are typically specified at 7 days for CSCSBC and the lab results were more consistent than the results from field cores. These companion cylinders strength and MOE results are shown in **Figure 8** also superimposed on the *NHI-05-037* relationships. The range of structural coefficients based on compressive strength was 0.155 – 0.352 (0.233 average). The correlation to MOE resulted in a layer coefficient between 0.119 – 0.508 (0.326 average). These results were based on 7-day breaks; therefore, they provided a more reasonable range of structural coefficient values. Using both the 7-day and 63-day values, the minimum specified compressive strength for CSCSBC required by ARDOT (750 psi) results in a minimum structural coefficient of *0.21*. This seems to be a rational lower bound for the structural coefficient for CSCSBC based on the testing reported here and the minimum compressive strength required [9].

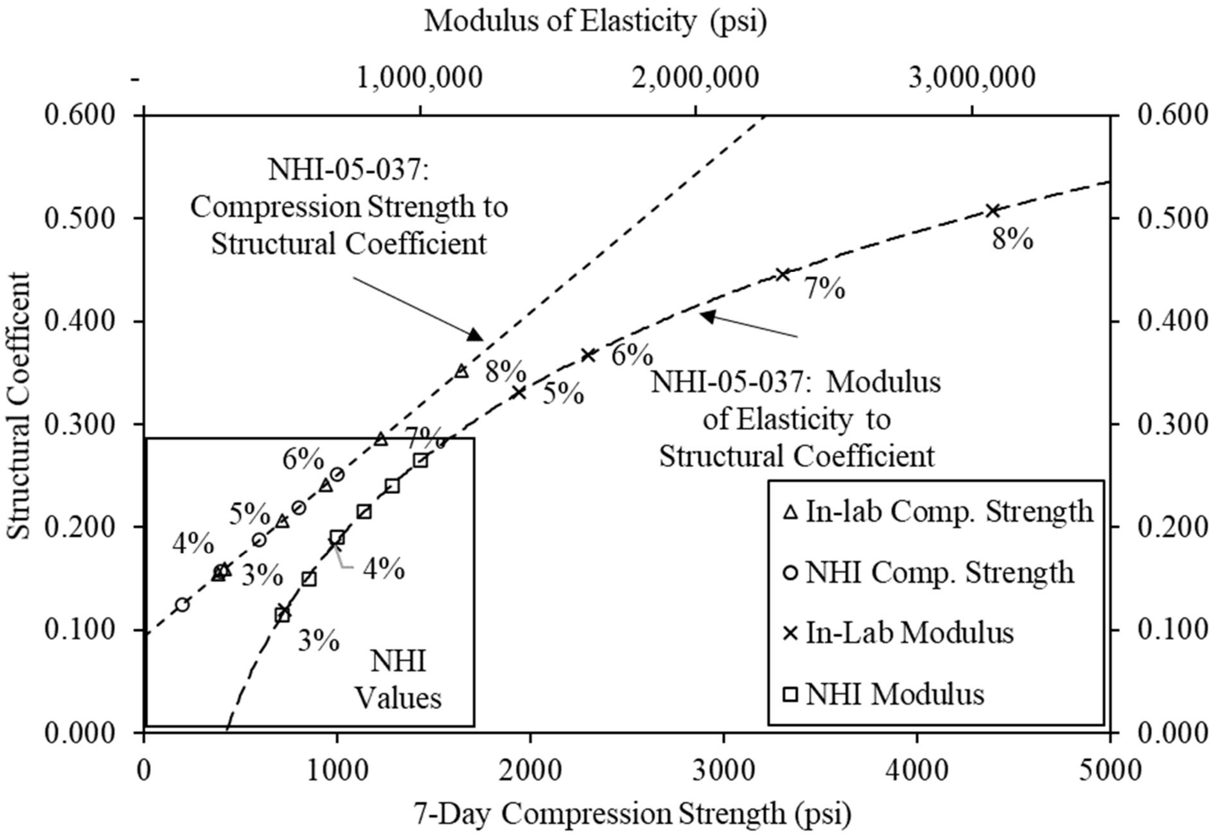

Figure 8: Model of *NHI-05-037* structural coefficient values based on 7-day testing

The relationship in **Equation 8**, based on the results from **Figure** 8, can also be used to calculate a structural coefficient. The ARDOT recommended minimum 7-day compression strength of *750 psi* results in a $a_2 = 0.21$. This relationship had good agreement with the compressive strength of lab samples tested in this research. It is recommended that **Equation 8** is bound between $a_2 = 0.21 – 0.30$. This is only to keep a conservative structural coefficient value. This occurs between *750 – 1,300 psi* compression strength.

[Equation 7] $a_2 = 0.0002 * \sigma + 0.0935$

$Where\ \sigma = Measured\ Compressive\ Strength\ of\ Material$ (psi)

A comparison was made between the results here and the practice in other states for the layer coefficient. This comparison is shown in **Figure 9**. These layer coefficients were taken from an AASHTO 1972 survey, state design directives, and technical reports [8]. The black bars on the left are the calculated values from the recommendations in this paper, the grey bar is the average structural coefficient recommended by other states, the next two bars are the structural coefficient values of class 7 and soil cement material recommended in Arkansas, and, finally, the rest are CSB structural coefficient values from other states [6, 8, 17, 18]

Based on these comparisons, any value within the range of *0.125 – 0.30* would be within the range practiced around the country, and it is by no means unusual to use a layer coefficient greater than *0.20* for CSCSBC.

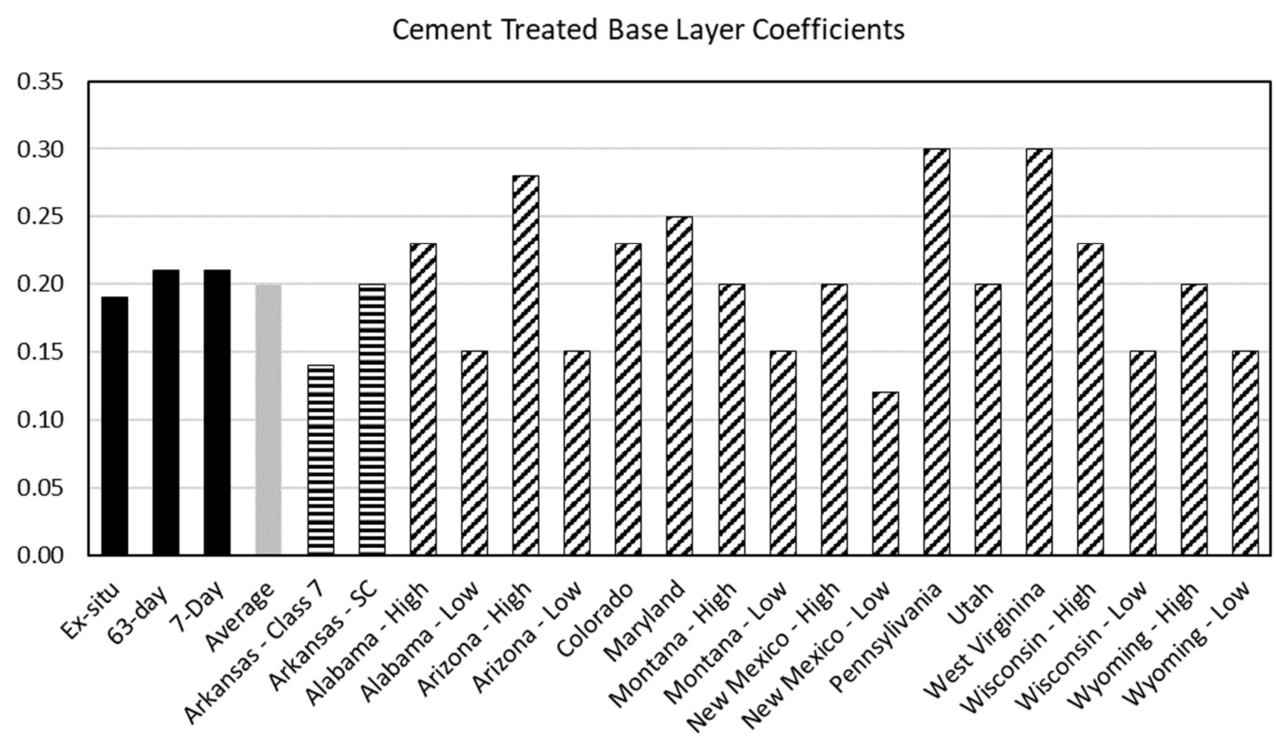

Figure 9: Comparison of cement treated base structural coefficients [6, 8,17, 18]

5.2 Recommendations for MEPDG

For MEPDG, MOE and Poisson's ratio are the pertinent design inputs for CSCSBC. Poisson's ratio proved difficult to measure reliably for this material, but MOE results were obtained from lab-made samples and field cores. **Table 10** showed results of the lab specimens tested at 7-days of age for compression strength, MOE, and Poisson's ratio. **Table 9** showed the MOE and compression strength results for field cores. The average values of modulus of elasticity show a consistent increase for each cement content, with a standard deviation of *268,203 psi*. Based on the testing shown here, it is safe to assume a CSCSBC layer with the minimum ARDOT compressive strength of *750 psi* should exceed *1,000,000 psi* MOE. The average MOEs from 63-day and 7-day testing were *2,234,700 psi* and *1,586,500 psi*, respectively. Poisson's ratio on the other hand had a large range (standard deviation of *0.12*) and matched closely with the MEPDG suggestions. **Table 11-7** in the MEPDG practice manual provides an estimate of Poisson's ratio as *0.10 – 0.20* and a modulus of elasticity as *1,000,000 psi* [3]. Based on testing reported here, these seem to be rational values to use for MEPDG designs. It may be warranted to use a larger value of MOE if the cement content is increased for added strength. In this case, the relationship given in **Equation 8** gives a strong estimate of MOE for the cement contents tested here.

[Equation 8] $E = 51{,}200{,}000 * p - 1{,}200{,}000$

$Where: E = Modulus\ of\ Elasticity\ (psi)$

$p = cement\ content, decimal$

6.0 CONCLUSIONS

The purpose of this testing was to provide a structural coefficient value and an accurate spring constant that can be used for CSCSBC in design. The recommendations are based on lab testing, field testing, and correlations reported in literature.

For flexible pavements designed by 1993 AASHTO, a minimum layer coefficient ($\boldsymbol{a_2}$) is recommended to be **0.21.** Higher values of cement or compression strength would result in higher structural coefficient values, and so a recommend $\boldsymbol{a_2}$ range would be **0.21-0.30**. This should be acceptable for any CSCSBC layer that meets ARDOT's minimum strength of *750 psi* at 7-days**.** As mentioned before, this is the average value reported by other states, and agrees with layer coefficient values based on in-lab 7-day strength testing. Achieving a *750-psi* compressive strength for 7-day breaks is achievable and results in a a_2 of *0.21* when cement content is higher than 4-5% based on *NHI-05-037* relationships and lab testing of compressive strength. A higher compressive strength CSCSBC layer should result in a higher structural coefficient, therefore an equation was proposed to calculate a layer coefficient if the measured compressive strength is known.

For rigid pavements designed by the 1993 AASHTO procedure, a $\boldsymbol{k_c}$ of ***2,000 psi/in*** was recommended. This is an upper bound in some of the AASHTO guidance but agreed well with StPT tests performed as part of this research and with back-calculations based on CSCSBC material properties. This increase in *k* should provide a reduction in pavement thickness of around *1 in*. for most projects (based on the sensitivity analysis in 2.2.2).

To use the MEPDG design philosophy, a modulus of elasticity and a Poisson's ratio for CSCSBC are needed. Based on the results in section 5.2, an equation was developed to provide a modulus of elasticity based on the cement content, given in **Equation 10**. A minimum MOE of *1,000,000 psi* is appropriate for all CSCSBC with a compressive strength above *750 psi*. The Poisson's ratio is suggested to be chosen from the *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice* [3], as most of the results of **Table 5** are consistent with **Table 11-7. Table 11-7** states a suitable value of Poisson's ratio for most projects is ***0.1*** to ***0.2.***

7.0 REFERENCES

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8.0 APPENDIX

Table 11: Modulus of subgrade reaction by depth of base and modulus of elasticity

Modulus of Subgrade Reaction, *k* (psi/in)			
Depth of base (in)	**Modulus of Elasticity of Subbase (psi)**		
	30,000	200,000	1,000,000
6	424	588	775
9	465	711	1,019
12	509	835	1,269
15	554	958	1,526

Table 12: Depth of concrete required for different modulus of subgrade reactions

Depth of Required Concrete (in)			
Depth of base (in)	**Modulus of Elasticity of Subbase (psi)**		
	30,000	200,000	1,000,000
6	13.49	13.29	13.10
9	13.42	13.14	12.86
12	13.36	13.01	12.65
15	13.29	12.89	12.45

Figure 10: CSCSBC strip set-up and construction

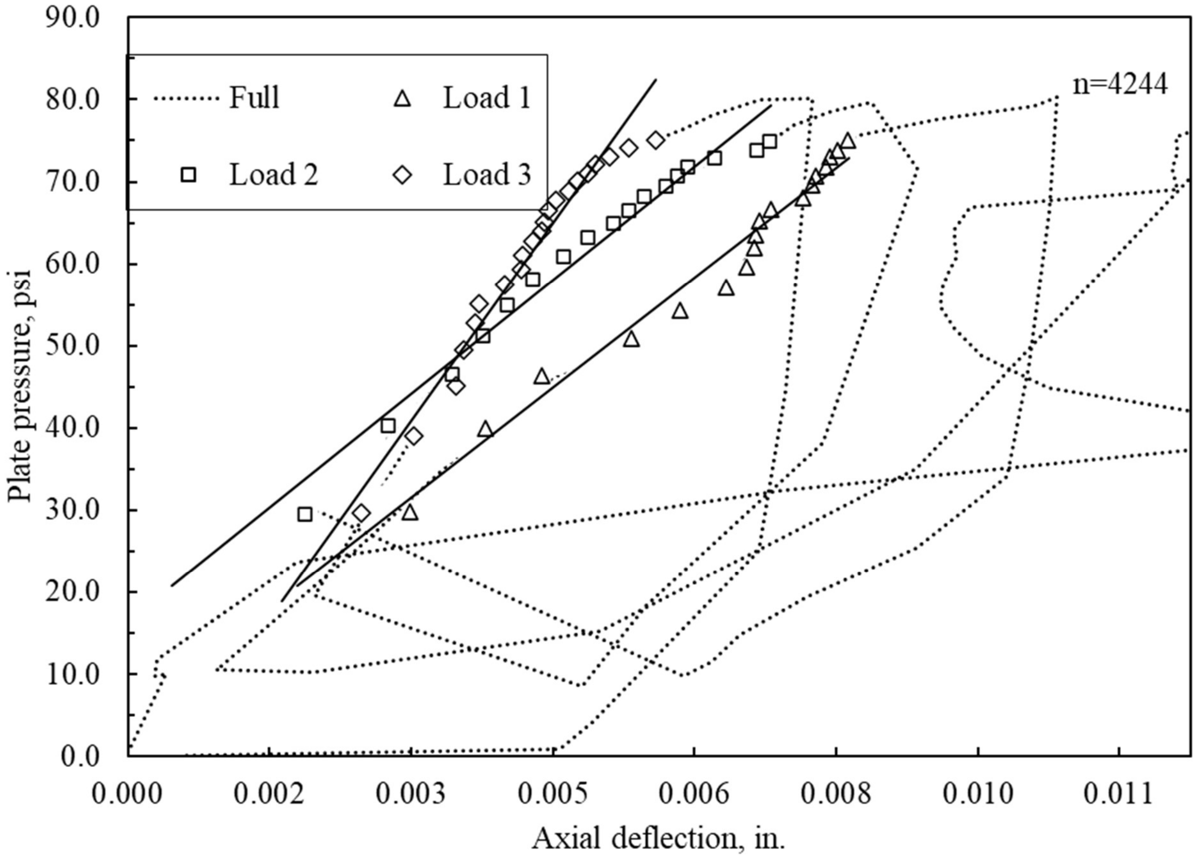

Figure 11: Static plate load test data from 3% cement content

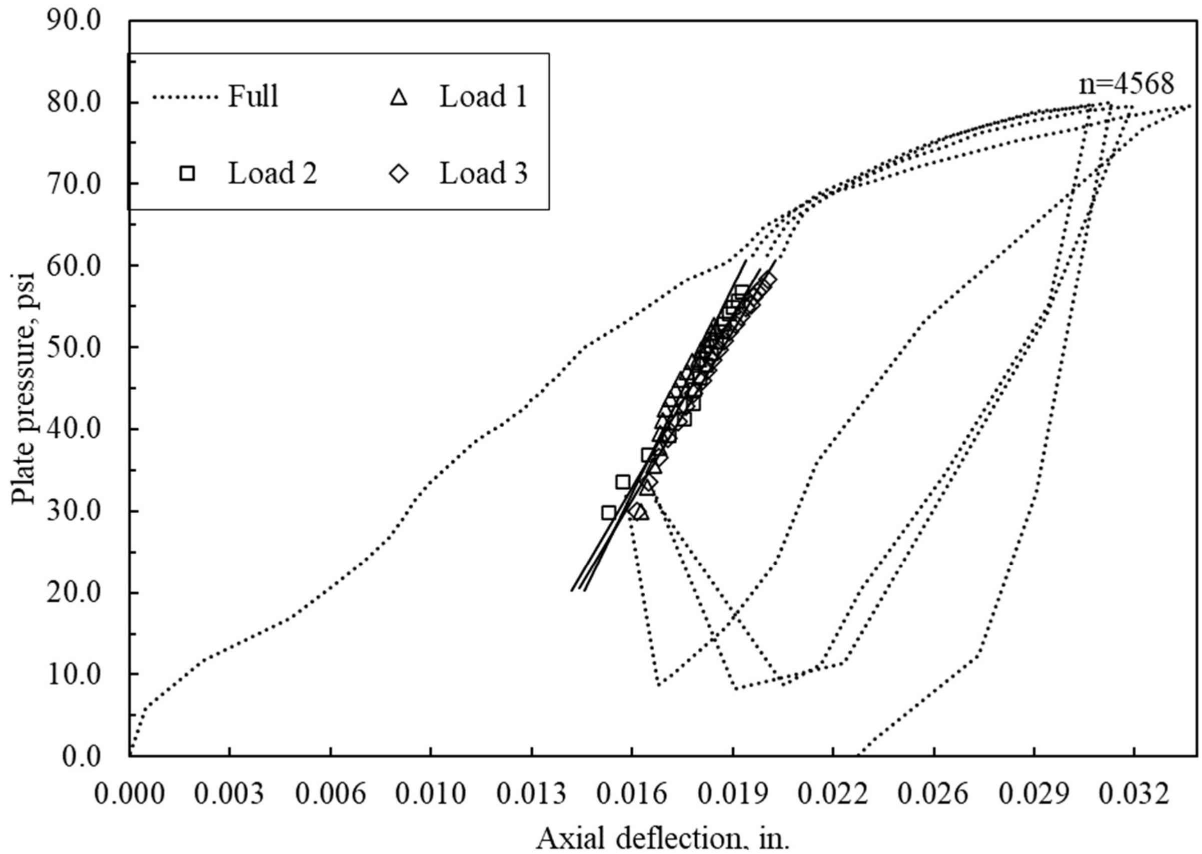

Figure 12: Static plate load test data from 4% cement content

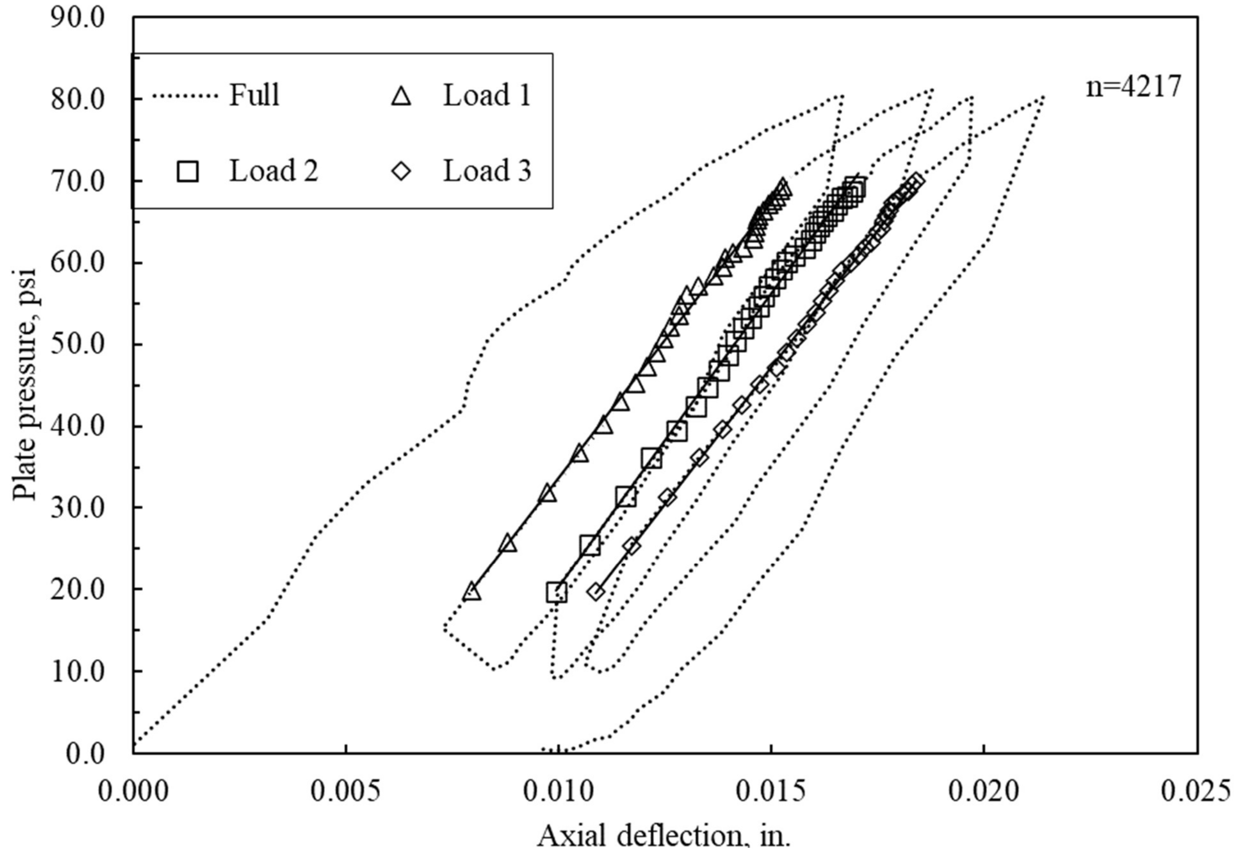

Figure 13: Static plate load test data from 5% cement content

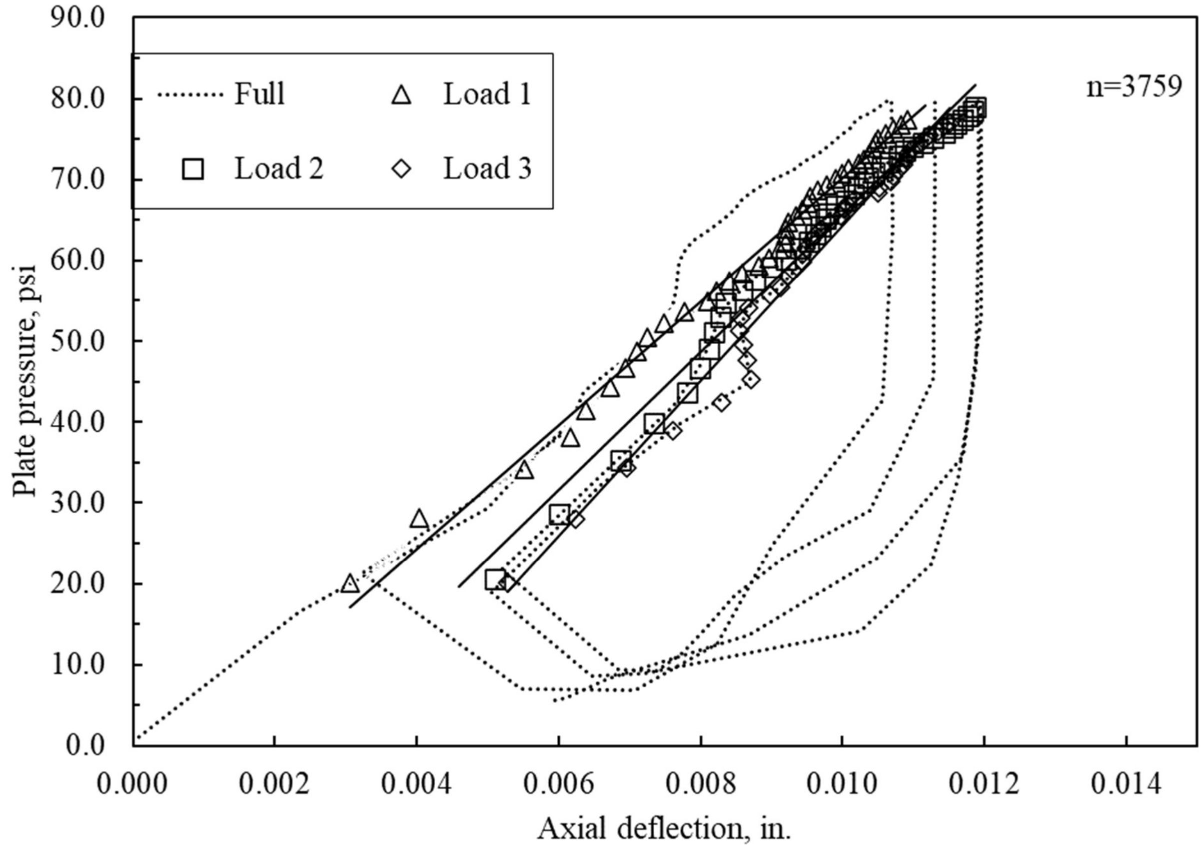

Figure 14: Static plate load test data from 6% cement content

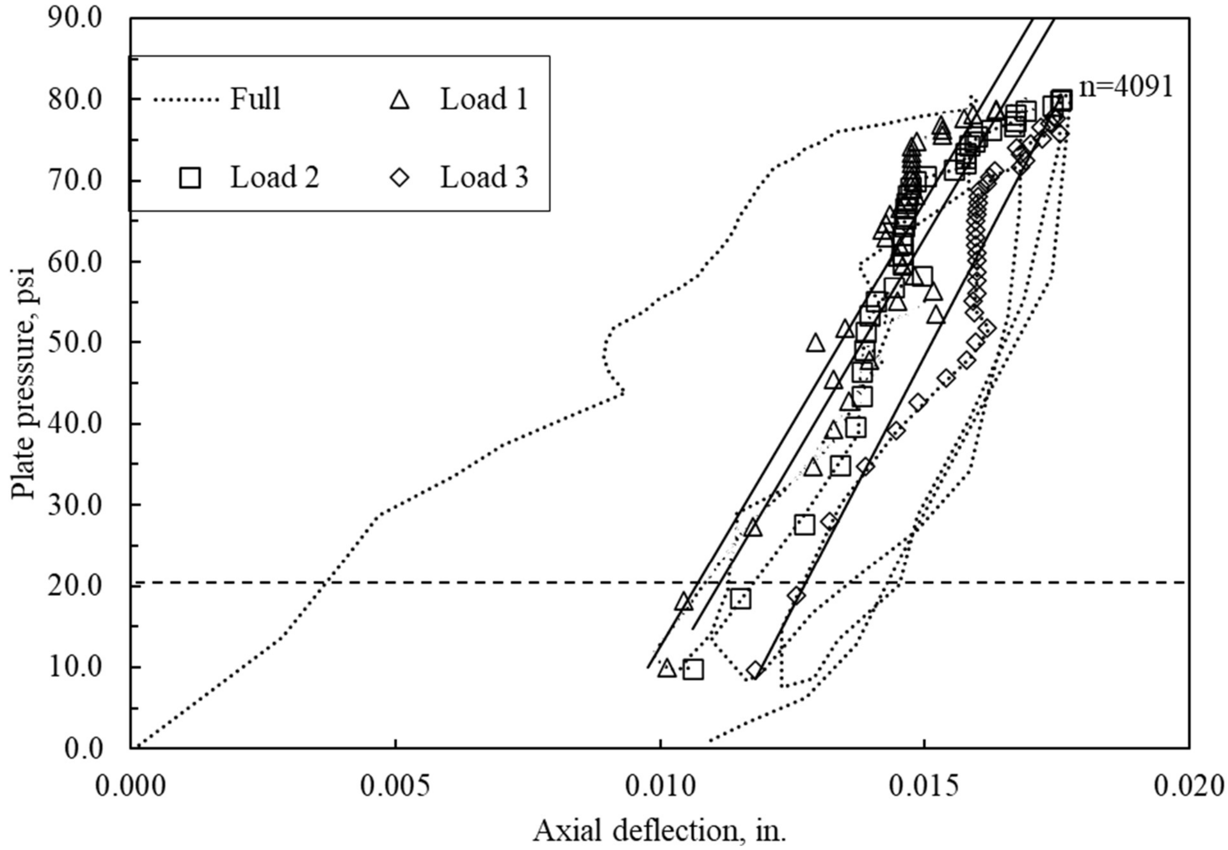

Figure 15: Static plate load test data from 7% cement content

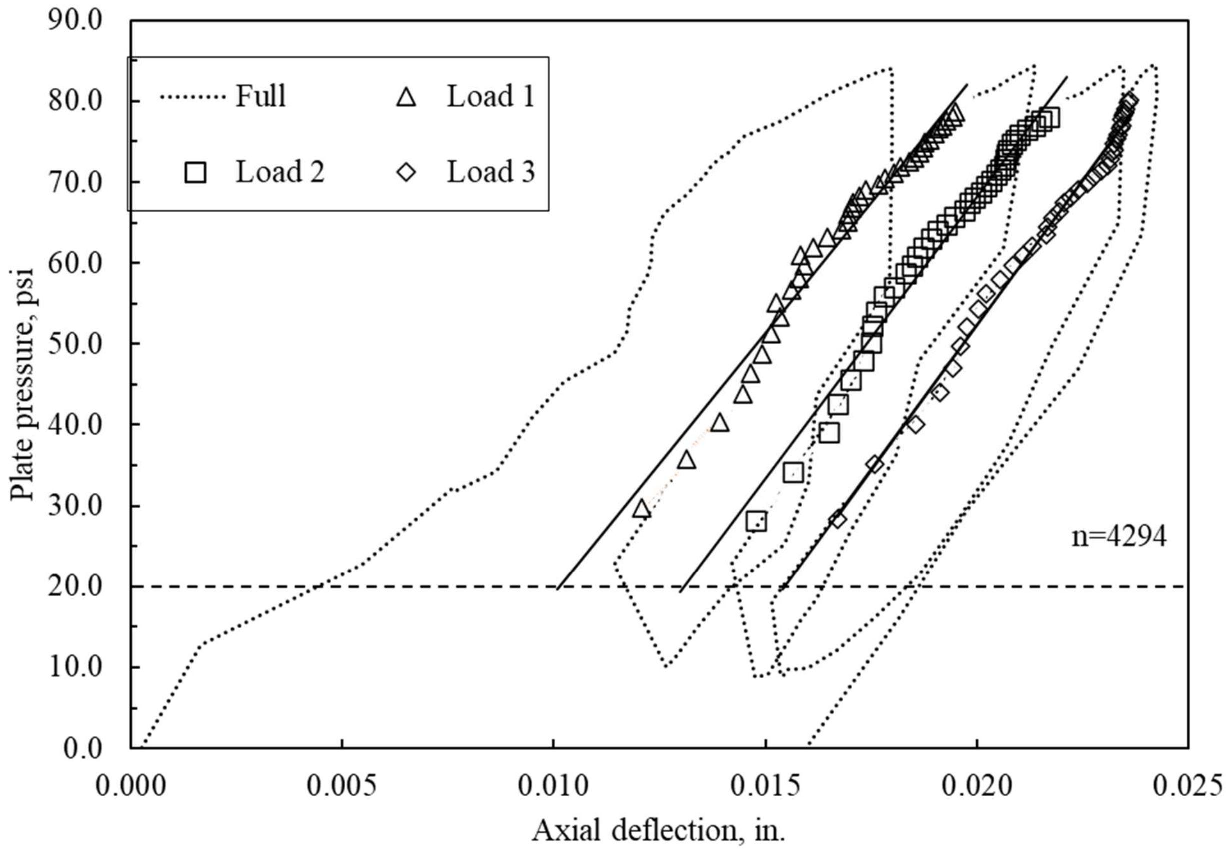

Figure 16: Static plate load test data from 8% cement content

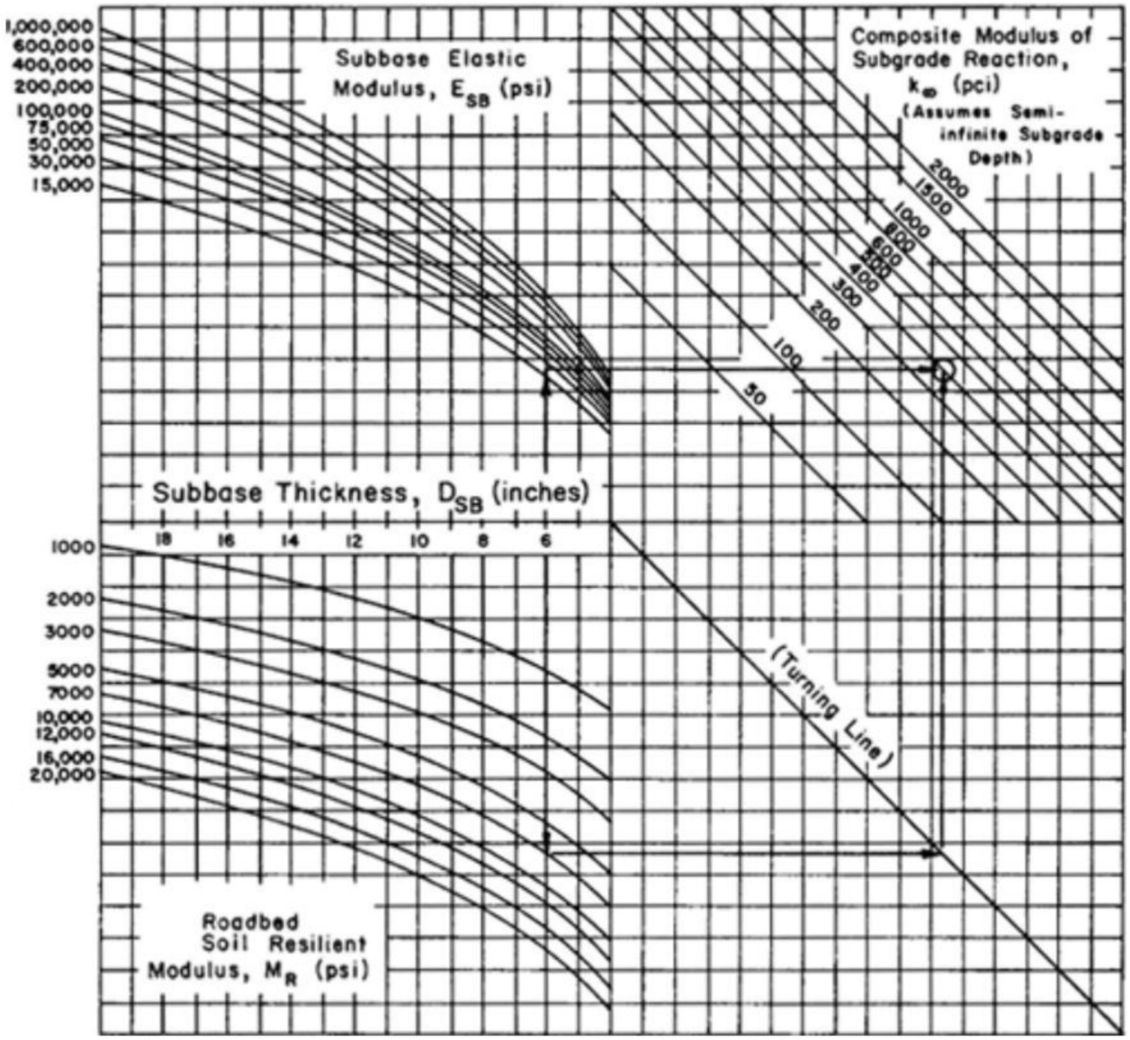

Figure 17: Composite modulus of subgrade reaction model [5]